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FLOW OVER REEFS AND STRUCTURES BY WAVE ACTION

BY

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Abstract

The discharge of water caused by wave action on reefs and structures
with impermeable uniformly sloping sides was measured for various elevations of
crest with respect to the still-water level. The experiments were completed for
side-slopes of 1:2 and 1:3 and for smooth and rough surfaces of the barrier. The
wave steepness, H/L, and the relative depth, d/L, were used as parameters and di-
mensionless plots were made for the discharge, (QT/H^2), as a function of relative
elevation, h/H, of the crest of the barrier.

Introduction

The crests of natural reefs generally are at or below the mean tide level.
It has long been observed that waves breaking over a reef will "pump" water over
the reef -- this water then usually flows along the lee side of the reef and re-
turns to the open sea through a gap or inlet. In times of "high" wave action
the currents through such inlets may be relatively large and even make naviga-
tion of surface craft quite hazardous.

In the case of man-made structures such as levees and dikes, the crests
are located well above the maximum expected high water. Extreme tides, wind
tides, or flood conditions, however, may raise the water stage to such a level
that wave action on the windward side might cause wave run-up to be excessive
and pump an appreciable quantity of water over the structure and thereby create
a flood condition in the area protected by the structure. A disaster of this
type occurred in Holland and Great Britain in February 1953 where the loss in
property and human lives was enormous. Such structures must be designed to
withstand not only the static, dynamic and erosive forces, but the crest eleva-
tions also must be sufficiently high to prevent overflow of water by wave action.

There appears to be little data available to permit a prediction of wave
run-up on shore protection structures under a wide variety of conditions; the
available information indicates, however, that wave run-up might amount to twice
the wave height or sometimes even more (Granthem, 1953). Considering high tide
ranges for some areas, and considerable wind tides (Keulegan, 1951; and Corps
of Engineers, 1951), it is possible that very high structures may be necessary
for some localities in order to prevent overtopping. Such high structures
usually are very expensive and a reduction of design height by only a foot might
result in considerable savings in construction costs.

As in the case of wave run-up, there is little information available
on the quantity of water that might be pumped over various types of structures
by wave action. The first experiments in this direction were completed by

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Saville and Cladwell (1953) for a vertical faced seawall. The results probably
will not apply for other types of structures. To fill this gap in our knowledge,
the series of tests described below were performed in the Wave Research Lab-
oratory of the University of California. The tests were conducted on structures
with a uniformly sloping impermeable face. Two different side-slopes were in-
vestigated (namely 1:2 and 1:3) under a variety of wave conditions. The results
are given as dimensionless plots of the variables involved.

Laboratory Equipment and Procedure

The experiments were conducted in a glass-walled wave channel 1 foot
wide by 3 feet deep by 60 feet long. Waves were generated by a flapper type
generator mounted at one end of the channel. Both the amplitude and the period
of the flapper movement were adjustable. The period could be varied between
approximately 0.4 seconds and 2 seconds, and the maximum wave height was about
0.5 foot. The water depth during all experiments was 1.69 feet.

The model was located approximately 45 feet from the wave generator and
was built in the form of a sheet-iron box as shown in Fig. 1. The front end
of the model, representing the structure, consisted of a sloping board. The
slope of the board could be varied by rotating it around point A (Figure 1),
and the crest elevation of the structure could be placed at various positions
above or below the still-water level. Special care was taken to seal all the
cracks between the structure and the channel walls, so that water circulation
could not occur. The roughness of the structure was varied by using different
plates fastened on the surface of the sloping board. The smooth surface was
represented by a smooth aluminum sheet, while for a rough surface transverse
wood strips were fastened to the aluminum sheet (see Figure 2). The relative
roughness was found to be equal to 0.036 foot for the smooth surface and 0.13
foot for the rough surface (Johnson, 1944).

Water which was pumped over the structure by wave action was caught in
a measuring tank, and the discharge was determined gravimetrically. The tank
was arranged so that the water passing over the structure could be diverted
either into the measuring tank or into a waste area. The arrangement was ne-
cessary to permit the measurement of discharge for any chosen wave group or
for a fixed length of time. The handle of the diversion flap was arranged so
that a contact to an electrically operated clock was closed the moment the flap
was opened to the measuring tank and the contact out again when the flap was
closed. This arrangement enabled a very accurate time measurement. The clock
was graduated to give a least reading of 1/100 of a second. The water in the
waste tank was returned to the channel at the backside of the model by means
of an auxiliary pump which operated continuously. This water could return
again to the main channel and thereby maintain the depth of water therein a
constant. A wave absorber was placed between the structure and the measuring
tank so that the flow over edge C (Fig. 1) would be more regular and not af-
fected by the turbulence of the breaking waves. The absorber also was par-
ticularly important for the case where the edge B of the structure was below
the still water level.

The wave heights and periods were measured about two to four wave
lengths seaward of the structure. Wave heights were measured by a double wire
resistance element and a Brush recorder.

Procedure: The structure was given a desired slope, θ, and the edge, B, set to a certain height, h, above (or below) still-water level. The distance, h, was taken as positive when the crest of the structure was above the still-water level, and as negative when below the still-water level. The average value for h was obtained by measuring the elevation with a point gage at at least three locations across the channel. In order to permit equilibrium conditions to exist after starting the wave generator, the water pumped over the structure by the first three or four waves was diverted to the waste area and then the measurement of water pumped was made of the following 3 to 10 waves according to the capacity of the measuring tank. Special care was taken to limit the measurement to the time before reflected waves from the structure could reach the generator and return to the structure. After the wave generator was stopped, the volume of the pumped water, the number of waves acting on the structure and the duration of the measurement as obtained from the electrically operated clock were recorded. The measurements were repeated five times for each condition. In general there was very little difference in measurements between the runs made under the same conditions. The wave heights and periods were obtained from Brush records, disregarding the first three or four waves and those which occurred after reflection effects became noticeable. By examining the wave records it was relatively easy to recognize the point where the reflection from the structure started to change the wave characteristics. The wave heights and periods as used represent the wave condition in the given depth of water as though no structure was present. A set of experiments usually was started with the crest of the structure approximately one wave height below the still-water level, and the crest was raised for the following runs in increments of approximately 0.05 ft. until a height, h, was reached where no appreciable overtopping of the structure occurred. Usually 40 to 50 single runs were completed for each set of experiments. Before starting each succeeding run the water surface was permitted to quiet completely.

The general test procedure was to select a given wave condition (H and T held constant) and then measure the discharge of water pumped over the structure for various elevations, h, of the crest. Tests were conducted under the following wave conditions:

1. For a constant wave steepness H/L of about 0.055, tests were made with d/L values of 0.339 and 0.589. The discharge passing over the structure was determined for several relative crest elevations, h/H.

2. For a constant relative depth d/L of 0.236, tests were made with wave steepnesses H/L of 0.0108, and 0.0182, and 0.0293. Here again the discharge passing over the structure was determined for several relative crest elevations, h/H.

For each of the above wave conditions tests were made with face slopes of 1:2 and 1:3 and with both smooth and rough surfaces. A summary of test conditions is given in Table 1. The experimental data from the various runs are plotted in Figures 3 and 4.

TABLE 1

Run No.	Slope of Structure	Surface of Structure	Depth of Water d, ft	Wave Period t sec.	Wave Length L ft.	Height H ft.	H/L	d/L	Maximum Q ft³/sec/ ft.	Max. "overtopping per crest of wave per ft. of structure Q_cr	Maximum QT/H² *	Maximum Uprush h/H
35-84	1:3	Smooth	1.69	0.75	2.87	0.16	0.0558	0.589	0.0200	0.0153	0.60	1.05
35-84	1:3	Rough	1.69	0.75	2.87	0.16	0.0558	"	0.0160	0.0120	0.47	0.85
85-131	1:3	Rough	1.69	1.00	4.98	0.26	0.0522	0.340	0.0500	0.0500	0.74	1.10
132-173	1:3	Smooth	1.69	1.00	4.98	0.26	0.0522	"	0.0570	0.0570	0.84	1.60
174-213	1:3	Smooth	1.69	1.25	7.17	0.130	0.0181	0.236	0.0276	0.0346	2.05	2.40
214-242	1:3	Rough	1.69	"	"	0.130	0.0181	"	0.0227	0.0284	1.68	1.90
243-281	1:3	Rough	1.69	"	"	0.21	0.0293	"	0.0550	0.0686	1.56	1.55
282-321	1:3	Smooth	1.69	"	"	0.21	0.0293	"	0.0585	0.0732	1.66	1.90
322-349	1:3	Smooth	1.69	"	"	0.0775	0.0108	"	0.0113	0.0131	2.35	1.85
350-377	1:3	Rough	1.69	"	"	0.0775	0.0108	"	0.0096	0.0120	2.00	1.50
378-399	1:2	Smooth	1.69	1.25	7.17	0.0775	0.0168	0.236	0.0092	0.0115	1.92	1.40
400-427	1:2	Rough	1.69	"	"	0.150	0.0182	"	0.0068	0.0085	1.42	1.10
428-452	1:2	Rough	1.69	"	"	0.150	0.0182	"	0.0208	0.0260	1.54	1.50
453-474	1:2	Smooth	1.69	"	"	0.210	0.0293	"	0.0235	0.0294	1.74	1.80
475-500	1:2	Smooth	1.69	"	"	0.210	0.0293	"	0.0635	0.0795	1.30	2.14
501-520	1:2	Rough	1.69	"	"	0.260	0.0522	"	0.0492	0.0616	1.40	1.50
521-540	1:2	Rough	1.69	1.00	4.98	0.260	0.0522	0.339	0.0427	0.0427	0.63	1.14
541-556	1:2	Smooth	1.69	"	"	0.260	0.0522	"	0.0542	0.0542	0.80	1.45
557-573	1:2	Smooth	1.69	0.75	2.87	0.16	0.0558	0.589	0.0165	0.0128	0.50	1.18
574-584	1:2	Rough	1.69	"	"	0.16	0.0558	"	0.0142	0.0110	0.43	0.92

* The values obtained from Figures 3 and 4
as maximums from experimental curves.

Experimental Results

A consideration of the various terms shows that the following dimensionless groupings can be used to represent the relationship between the water passing over the structure and the other variables:

$$TQ/H^2 = f(h/H; d/L; H/L; \theta)$$

where

Q = water passing over the structure by wave action in cfs per foot of structure (ft^2/sec.)

T = wave period in seconds

L = wave length in feet

H = wave height in feet

h = elevation of the structure in feet above (or below) the still-water level.

θ = slope of the structure

To demonstrate the relative effect of the variables, the test data are summarized in Figs. 3 and 4. Figs. 3 a-b and 4 a-b show for slope 1:2 and 1:3 respectively, the rate of overtopping as a function of crest height for two values of relative depth and a constant wave steepness. Figs. 3 c-e and 4 c-e show the rate of overtopping as a function of crest height for three wave steepnesses and a constant relative depth.

The effect of the relative crest elevation of the structure above or below the still-water level is demonstrated in Figs. 3 and 4. Smooth curves have been drawn for all the various test conditions. In general the shape of all the curves is very similar. The maximum overtopping seems to occur when the crest of the structure is slightly above the still-water level with an average value of h/H approximately 0.2. From this point on, the amount of overtopping water decreases as the crest of the structure is placed further below the still-water level. These results apply naturally only for the conditions where the still-water level is at the same elevations on both sides of the structures, which is the case for most harbor protection structures, reefs, etc. The conditions where there is no water on the lee side of the structure would be the case of flow over a weir combined with surface wave action. This condition has been treated previously by Gibson (1930).

As can be seen in Figures 3 and 4, the roughness of the face of a structure has a considerable effect on the amount of overtopping water. At the maximum rate of overtopping, the rough surface results in an approximately 20 percent less amount than for the smooth surface. For higher values of h/H this percentage is even higher. In general, the empirical curves indicate that the rate of overtopping on a structure with relatively rough sides may be 20 percent lower than on that with smooth sides. In the case where the edge of the structure is below the still-water elevation, the roughness does not influence the results appreciably. It seems even that in this region the rough surface results in higher overtopping for some cases. This seems to be logical, for a rough surface prevents the backflow of water more than a smooth surface.

Relative depth of water: d/L is also a factor which would influence the rate of flow over a reef or structure by wave action. The steepness of

the waves approaching shallow water depends on the relative depth of the water, and so does the critical steepness given by the formula:

$$(H/L)_{critical} = 0.140 \tanh 2\pi d/L$$

A wave which is near the critical steepness for breaking can be easily "tripped" and made to break by any outside disturbance. The location of the breaking point was found to be an important factor in the overflow by wave overtopping. When the wave broke just over the edge of the structure, the discharge caused by wave overtopping was relatively high. On the other hand, when breaking occurred before the wave reached the structure, considerable energy was lost and the discharge by overtopping was relatively small.

Wave steepness: To compare the results obtained under various wave steepnesses, but with a constant d/L, the curves from Figures 3 c-e and 4 c-e were replotted in Figure 5. A comparison of the various curves shows that the maximum rate of overtopping generally was greatest for the waves of small steepness. This is more evident in Figure 6 where the maximum values of QT/H^2 as obtained from the curves in Figure 5 are plotted as a function of wave steepness, H/L. These curves indicate an increasing value for the maximum value of QT/H^2 as the wave steepness decreases. There appears to be a certain wave steepness which results in a maximum value of $(QT/H^2)_{max.}$, however, considerably more data are necessary to draw more definite conclusions. Figure 6 indicates also a higher discharge of water for a structure with a side slope of 1:3 than for one with a slope of 1:2.

Wave run-up: To supplement the previous work on wave run-up by Grantham (1953), data have been obtained from Figures 3 and 4. Wave run-up in this instance was equal to the elevation of the crest of the structure when the latter was just high enough to prevent overtopping. Thus, for the data shown in Figures 3 and 4 the wave run-up was computed from the relative crest elevation, h/H, at the point where the curve showed a zero discharge by overtopping. The computed values are plotted i. Figure 7 as a function of wave steepness, H/L, for slopes 1:2 and 1:3 and for smooth and rough surfaces of the structure. Figure 7 indicates that there is a critical wave steepness which results in the highest run-up of the waves. The wave steepness determines the breaking point, and when breaking occurs just at the edge of the structure the run-up is a maximum. When the breaking point moves seaward, the run-up decreases. When the waves do not break on the structure, the run-up decreases again with a decreasing wave steepness, and it is expected that it would reach the half wave height for very long waves (such as the tide). This is confirmed also by the fact that the curves in Figure 7 indicate a trend toward the value h/H = 0.5 for zero steepness. Figure 7 indicates also that there is a different critical steepness for different structures. For the slope 1:2, this critical H/L value is higher than for a 1:3 slope for the given wave condition.

As shown in Figure 7, the roughness of the structure also has an important bearing on the wave run-up. Thus run-up is about 30 percent higher for a smooth surface as compared with a rough surface.

Conclusions

Within the limits of the experiments on uniformly sloping impermeable

structures, it is concluded that:

1. For a given relative height of a structure and a given relative depth, the overtopping of water caused by wave action is largest for the structure with a smooth face.

2. The maximum rate of overtopping occurs when the structure is approximately 1/5 of the wave height above the still-water level, provided that the still-water level is at the same elevation on both sides of the structure. For a given relative depth and surface roughness, the maximum rate of overtopping decreases as the wave steepness increases.
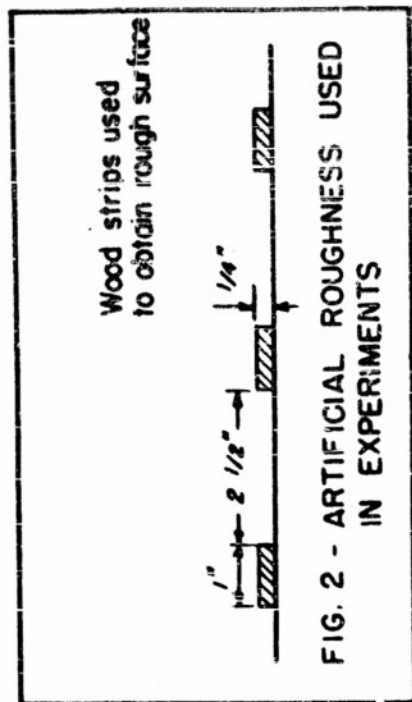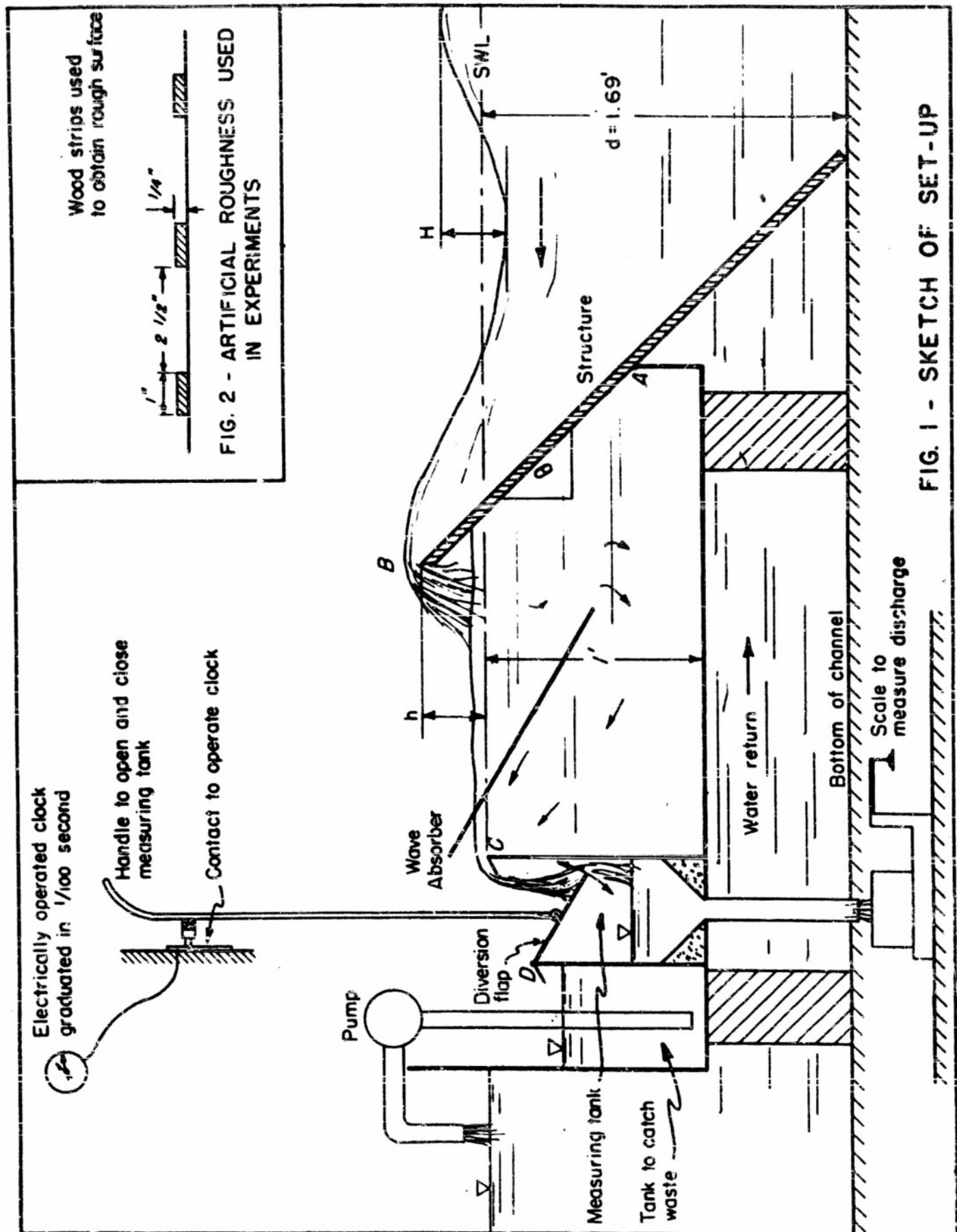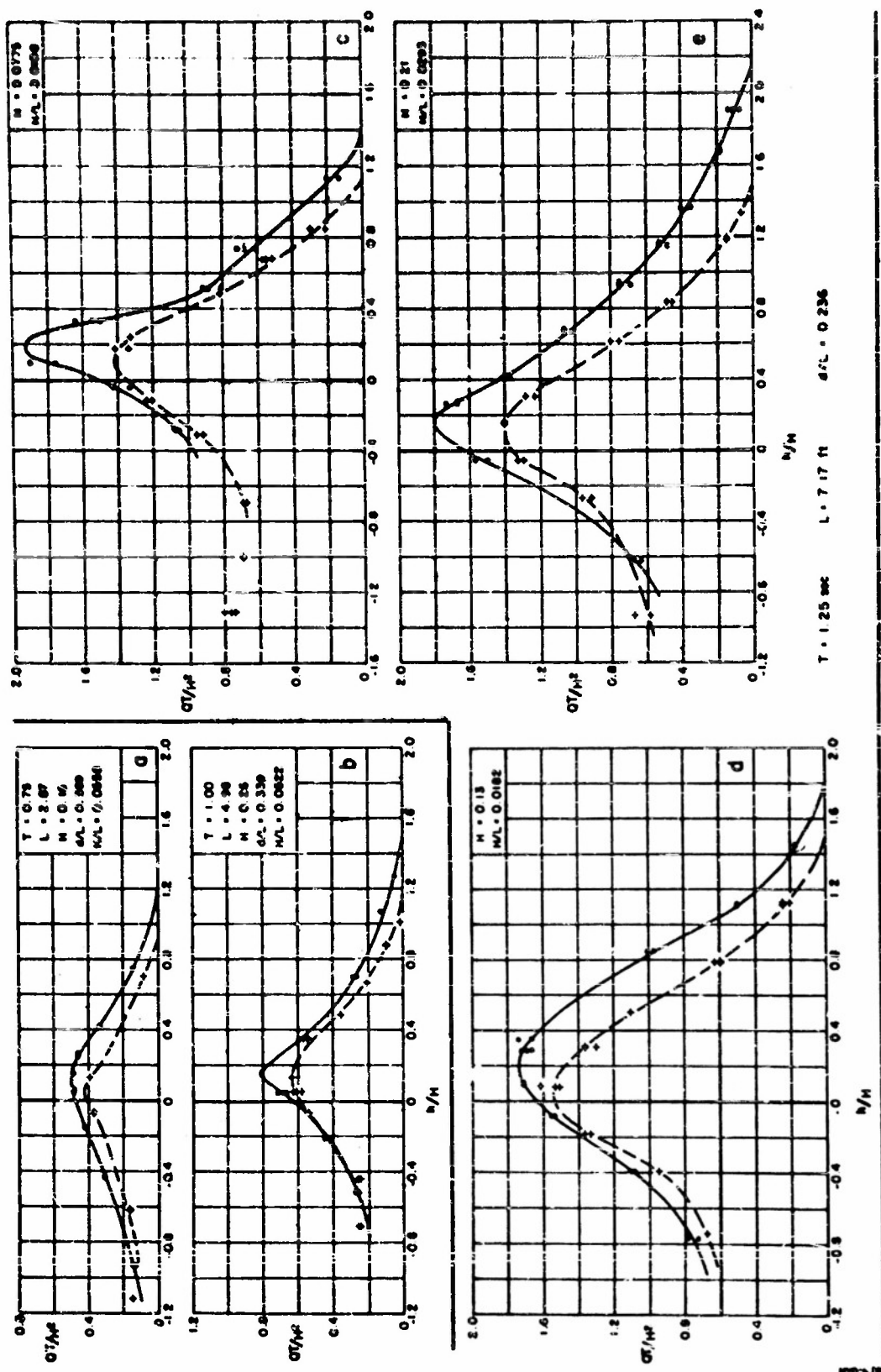
Acknowledgments

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References

CORPS OF ENGINEERS, U.S. ARMY Central and Southern Florida Project (1951) July 10, (Not for public release) especially pp 1-129 and on.

GIBSON, A.H. (1930) The effect of surface waves on the discharge over weirs. Selected Engineering Paper, No. 99, Inst. of C.E., London.

GRANTHEM K.N. (1953) A model study of wave run-up on sloping structures. Trans. Amer. Geophys. Un., vol. 34, No. 5, Oct. 1953, pp. 720-724.

JOHNSON, J.W. (1944) Rectangular artificail roughness in open channels. Trans. Amer. Geophys. Union, pp. 907-914.

JOHNSON, J.W., FUCHS, R.A., and MORISON, J.R. (1951) The damping action of submerged breakwaters. Trans. Amer. Geophys. Union, vol. 32, No. 5, Oct. 1951, pp. 704-713.

KEULEGAN, G.H. (1951) Wind tides in small closed channels, Jour. of Research of the Nat. Bu. Stand., vol. 46, No. 5, May 1951.

SAVILLE, THRONDIKE JR. and CALDWELL, J.M. (1953) Experimental study of wave overtopping on shore structures. Beach Erosion Board (Unpublished).

Wood strips used to obtain rough surface

FIG. 2 — ARTIFICIAL ROUGHNESS USED IN EXPERIMENTS

FIG. 1 — SKETCH OF SET-UP

HYD-6765

FIGURES 1 & 2
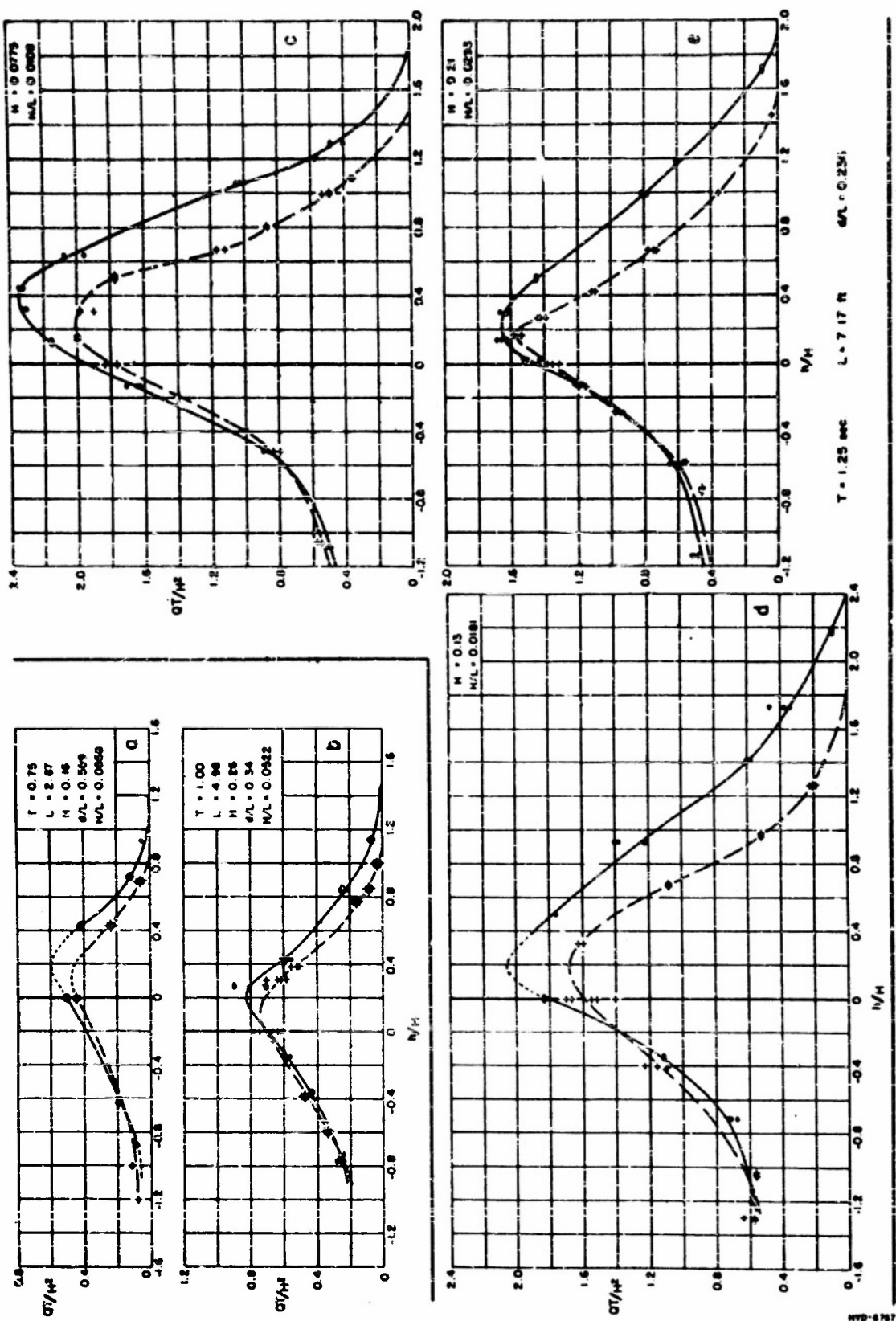
WAVE OVERTOPPING ON MARINE STRUCTURE AS A FUNCTION OF RELATIVE HEIGHT OF STRUCTURE

Side slope, θ = 1:2
Depth of water, d = 1.69 feet
——— Smooth surface   ——·—— Rough surface

FIGURE 3

WAVE OVERTOPPING ON A STRUCTURE WITH UNIFORMLY SLOPING IMPERMEABLE SIDES

Side slope, θ = 1:3   Depth of water, d = 1.69 feet

——— Smooth surface   —·—— Rough surface

FIGURE 4
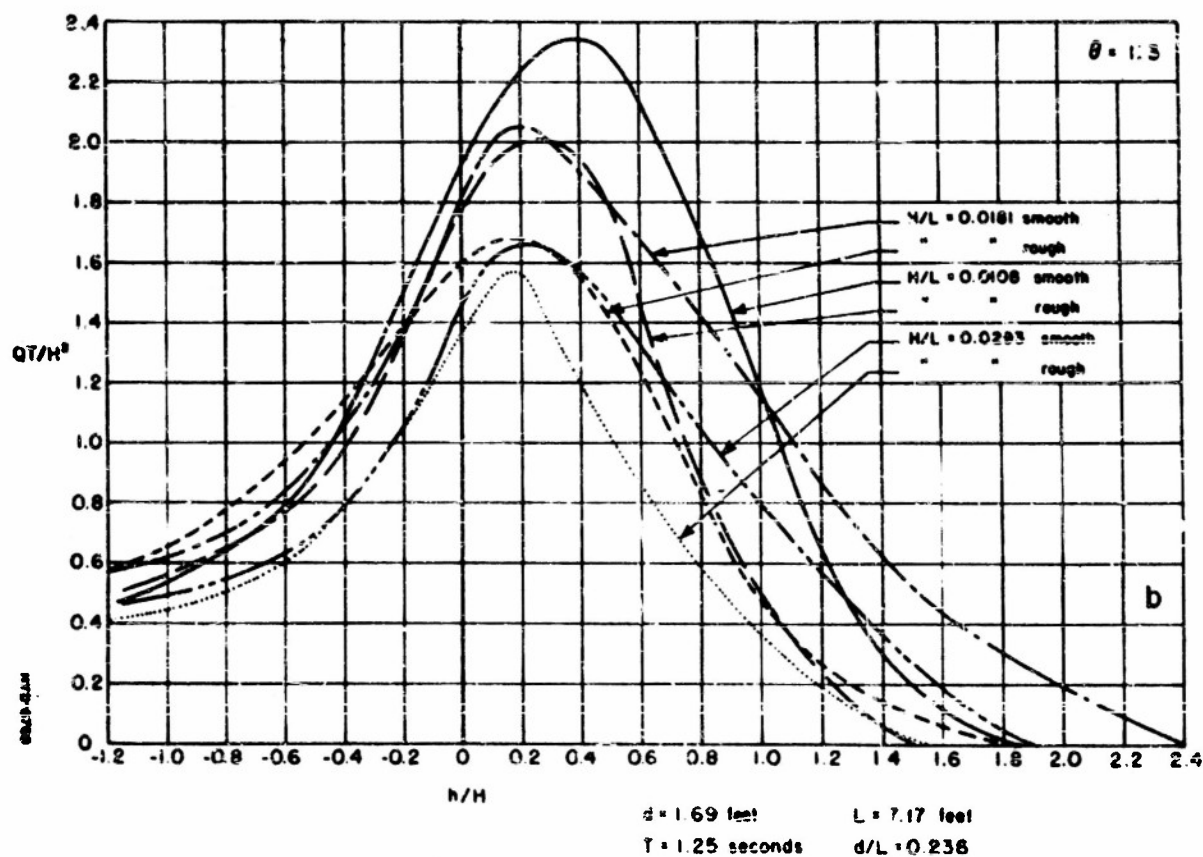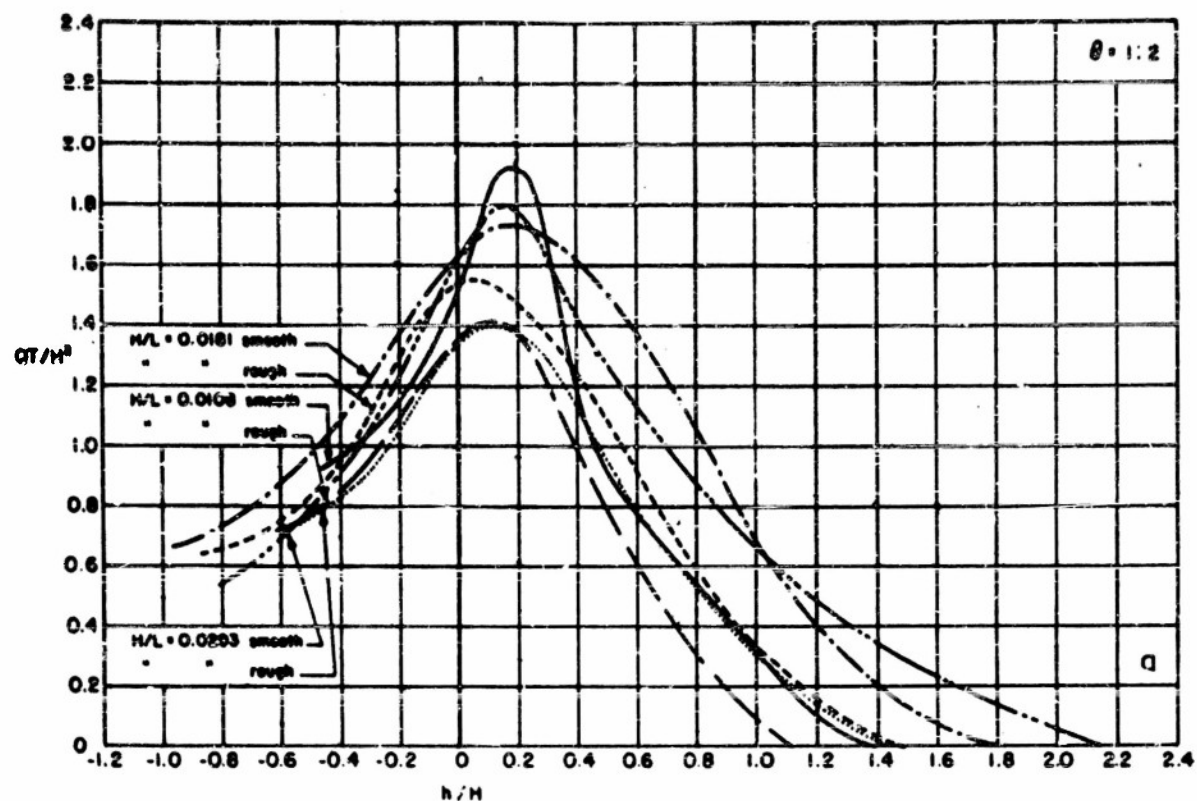
WAVE OVERTOPPING AS A FUNCTION OF HEIGHT OF STRUCTURE ABOVE STILL WATER
FOR CONSTANT d/L RATIO AND VARIABLE WAVE STEEPNESS

FIGURE 5

FIG. 6 - MAXIMUM VALUE OF QT/H^2 AS A FUNCTION OF WAVE STEEPNESS

FIG. 7 - WAVE RUSH-UP, h/H, ON SLOPING STRUCTURES AS A FUNCTION OF WAVE STEEPNESS
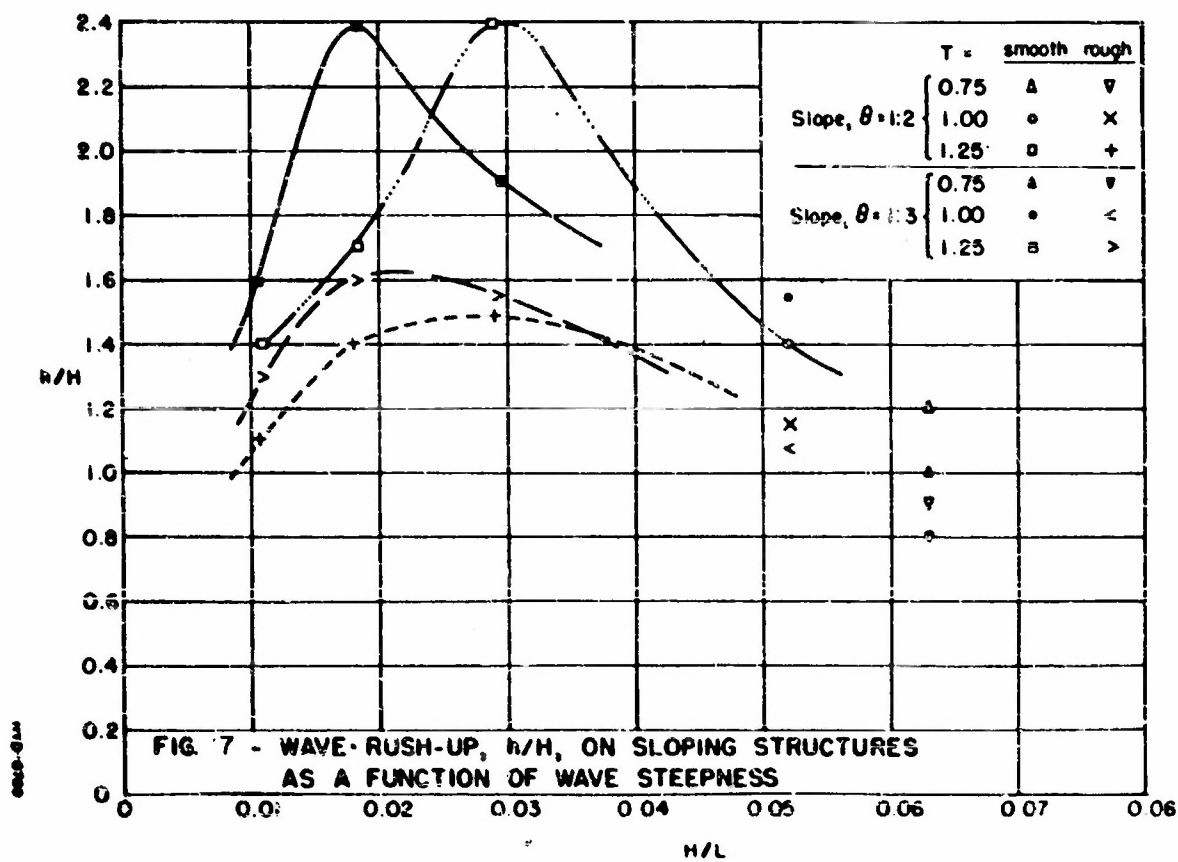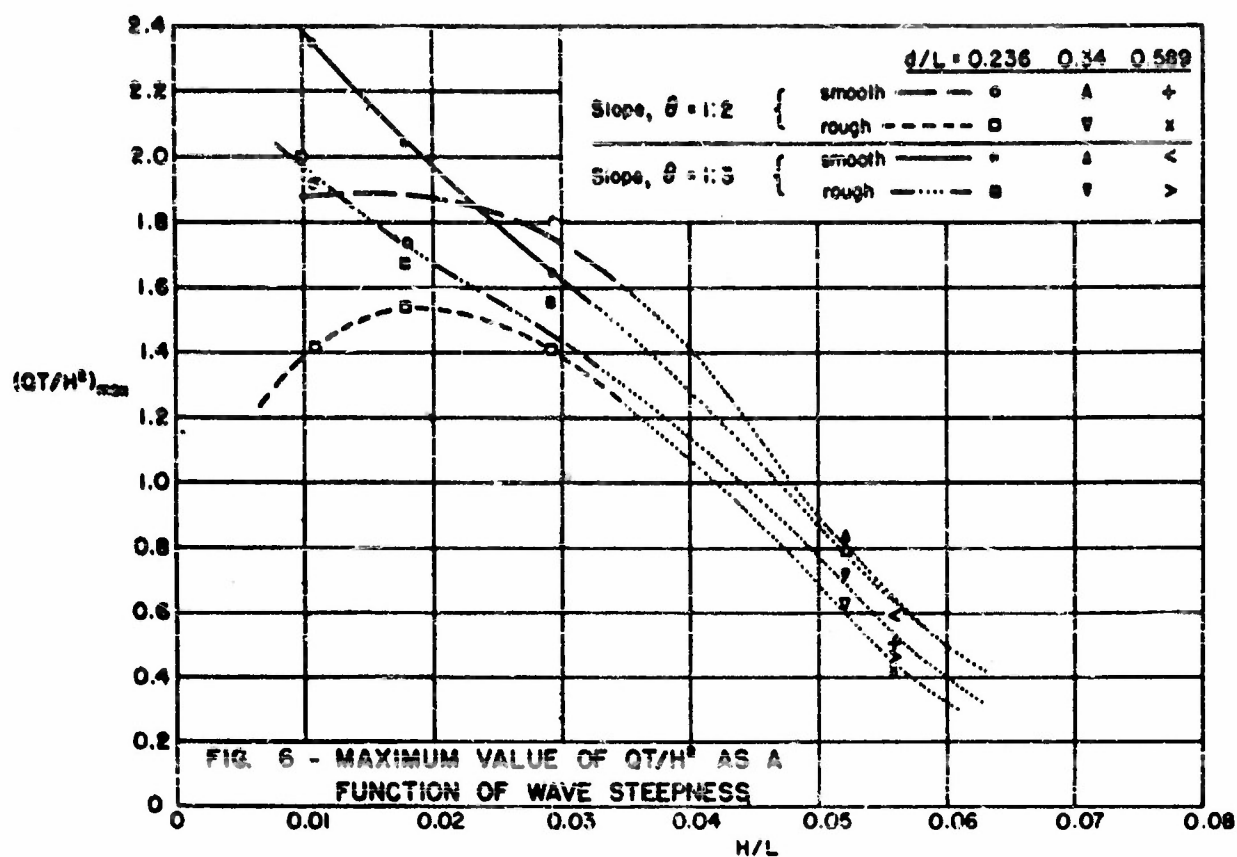
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